PUNCHING OF REINFORCED CONCRETE SLABS NUMERICAL AND EXPERIMENTAL ANALYSIS AND COMPARISON WITH CODES

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Abstract: *Nonlinear analysis programs are becoming increasingly popular in recent years as engineers attempt to more realistically model the behavior of structures. There has been some work regarding nonlinear FE analysis (FEA) of punching failure. Nevertheless, some additional work must be done in order to improve our knowledge about this phenomenon. This work presents some numerical modelling results of column/slab connections without punching shear reinforcement using ATENA 3D, in order to predict the punching failure behavior, including: failure loads, deflections and strains in the reinforcement steel. The nonlinear behaviour of concrete is one of the most important features of ATENA 3D, and is based on the real behaviour of concrete under tension and compression. The models used in this analysis are based on the smeared crack concept and the fixed crack model. A comparison between reliable experimental and numerical nonlinear results of slab-column connections subjected to punching load is presented and discussed. The results obtained are also compared with expressions given by ACI 318-08 and EC2-2004.*

1. INTRODUCTION

Flat slabs are widely used in many countries because of their economic and functional advantages. Although simple in appearance, a flat slab system presents a complex load bearing behaviour, especially in the slab-column connection. The punching resistance is an important subject in the design of flat slabs, frequently being the conditioning factor in choosing its thickness.

With numerical analysis it is possible to study a larger number of cases than with experimental tests, with lower cost and preparation time. The existing numerical models still do not present satisfactory results when dealing with the punching phenomenon. So, it is the purpose of this work to present results from a three-dimensional numerical analysis of reinforced concrete column/slab connections without punching shear reinforcement, modelled using ATENA 3D[1], in order to predict punching failure behaviour, including failure loads, deflections and steel bar strains. For this purpose, the numerical results will be validated against recent experimental work (Ramos[2,3] and Faria[4])and also compared with codes of practice. An accurate prediction of the behaviour of reinforced concrete slabs (RCS) is relevant not only to achieve a safe and economic structural design, but also to access the safety of old RCS which have not been designed according to the latest design guidelines. The FEA has been successfully used in the numerical analysis of reinforced concrete structures over the last thirty years. Nevertheless the prediction of the overall load-deflection response of RCS still poses several challenges because of its extremely complex nonlinear behaviour.

2. MODELS DESCRIPTION

The experimental work described in this paper consisted in the testing of five reduced scale reinforced concrete flat slab models up to failure by punching. Slabs AR2, AR9 and DF1 were 2300x2300 mm^2 with 100 mm thick, DF4 was 2300x2300 mm^2 with 120 mm thick and slab ID1 was 1800x1800 mm^2 with 120 mm thick. The punching load was applied by a hydraulic jack positioned under the slab, through a steel plate with 200x200 mm^2 in the centre of the slabs. Eight points on the top of the slab were connected to the strong floor of the laboratory using steel tendons and spreader beams (Figure 1).

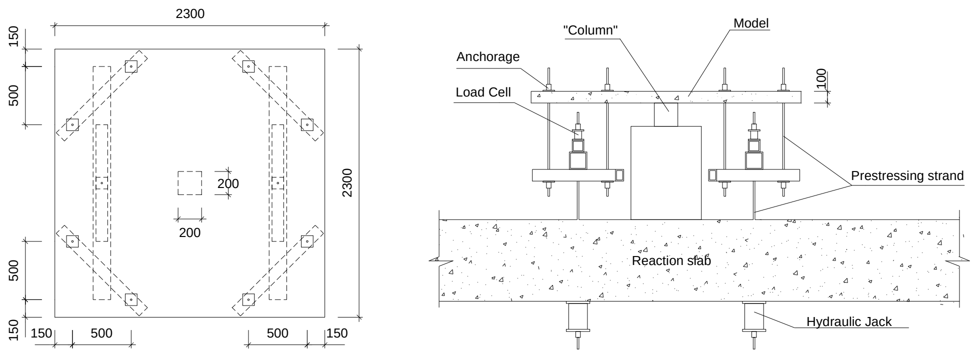

Figure 1: Models geometry (ex. model AR2)

They modelled the area near a column of an interior slab panel up to the zero moment lines. The bottom reinforcement consisted of 6 mm rebars every 200 mm in all slabs, in both orthogonal directions. In slabs AR2, AR9 and DF1 the top reinforcement consisted on 10 mm rebars every 60 mm and in slabs DF4 and ID1 the top reinforcement was 10 mm rebars every 75 mm, in both orthogonal directions. The effective depths of the top reinforcement was in average 80 mm for slabs AR2 and AR9, 69 mm for slab DF1, 88 mm for slab DF4 and 87 mm for slab ID1.

2.1 Materials Properties

To assess the strength of the concrete used in the production of the test specimens, compression tests on cubes of 15x15x15cm^3 were carried out ($f_{cm,cube}$). The results are listed in Table 1. This table also presents the values considered for the cylinder compression strength (f_{cm}) and for the axial tensile strength of the concrete (f_{ctm}), obtained with the relations presented in MC90[5]. The reinforcement steel tensile yielding (f_{sy}) and strength (f_{su}) used in the models is also presented in Table 1.

Model	$f_{cm,cube}$ (MPa)	f_{cm} (MPa)	f_{ctm} (MPa)	ϕ6		ϕ10	
				f_{sy} (MPa)	f_{su} (MPa)	f_{sy} (MPa)	f_{su} (MPa)
AR2	48.9	39.1	3.0	639	732	523	613
AR9	46.4	37.1	2.9	555	670	481	633
DF1	31.0	24.8	2.0	537	656	541	637
DF4	24.7	19.8	1.6	561	678	537	648
ID1	49.2	39.3	3.0	588	697	445	582

Table 1: Concrete and reinforcement steel properties.

2.2 Instrumentation

Load cells, for the vertical applied load, displacement transducers and strain gauges, glued to the top reinforcement steel, were used to monitorize the tests. The displacement transducers and strain gauges used in model DF4 are presented in Figure 2. The strains in the top reinforcement steel, the vertical displacements of the slab and the total applied load at each load stage, were measured continuously during the tests using a data acquisition system.

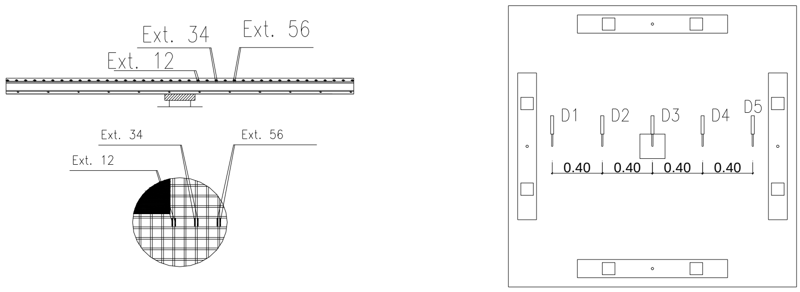

Figure 2: Model DF4 instrumentation: strain gauges (left), and displacement transducers (right).

3. EXPERIMENTAL/FEA RESULTS AND COMPARISON WITH CODES

3.1 Mesh discretization

The constant improvement of the computational capacities makes three-dimensional models more attractive, since it is possible to model various characteristics in any direction, and it is also suitable for the study of the spatial behaviour of the punching phenomenon. The calculations were made using the finite element program ATENA 3D[1] that, among other capabilities, is suitable for nonlinear static calculations. Figure 3 represents one of the models used in this study and an example of the screen output aspect. The numerical model uses three-dimensional isoparametric elements with eight nodes and only one quarter of the slab was modelled using symmetry. The punching failure zone was discretized with a fine mesh of brick finite elements, being the outer zone meshed also with brick finite elements, but with larger dimensions. The reinforcement steel was modelled with truss elements and the bond was considered through MC905 bond slip law. The load on the slab was introduced by deformation control. The models used in this analysis are based on the smeared crack concept and the fixed crack model.

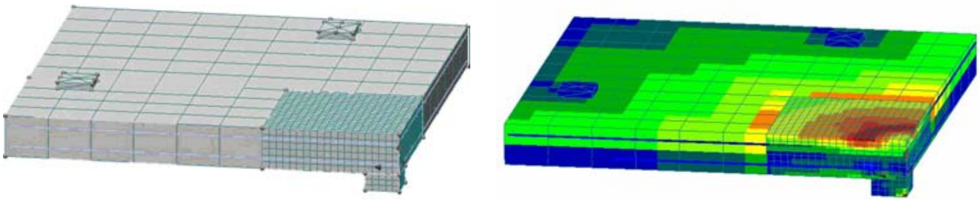

Figure 3: Example of a three dimensional mesh discretization (model DF4)

3.2 Load/displacement results

This section presents the results obtained for the displacements in the top of the slabs. In Figures 4 to 6 and Table 2 is presented a comparison between the displacements measured in the experimental tests and in the FEA, for several load levels (Table 2).

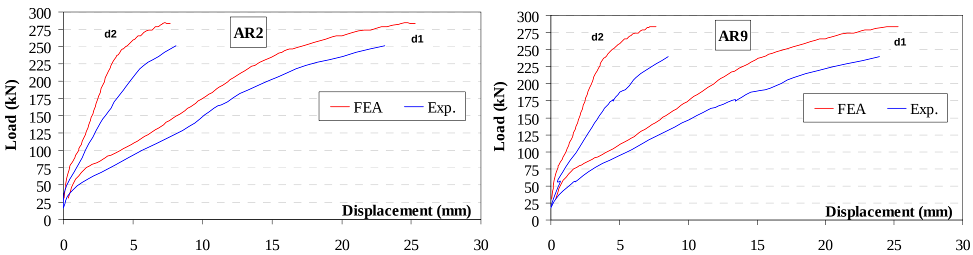

Figure 4: Load/displacement graphs for models AR2 and AR9.

In Table 2, d_1 represents the displacement that corresponds to the average of D1 and D5, while d_2 represents the average of D2 and D4 from tests, relative to D3 (see Figure 2). The average value of the ratio Exp./FEA for all the tested models is 1.23 (it varied between 0.84

for Model ID1 to 1.62 for Model AR9), witch gives a satisfactory agreement between the experimental results and the FEA predictions.

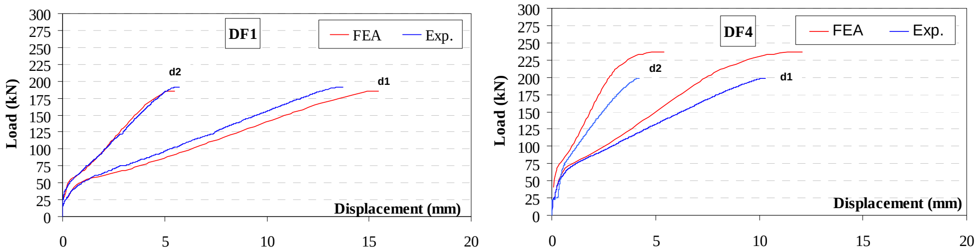

Figure 5: Load/displacement graphs for models DF1 and DF4.

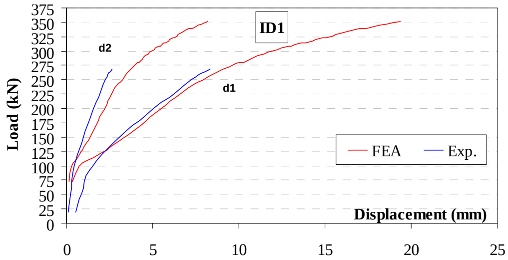

Figure 6: Load/displacement graph for model ID1.

Model	Load (kN)	d_1 (mm)				d_2 (mm)			
		Exp.	FEA	Exp/FEA	Aver.	Exp.	FEA	Exp/FEA	Aver.
AR2	100	5.69	3.78	1.51	1.35	1.61	1.00	1.61	1.65
	150	10.14	7.98	1.27		3.03	2.01	1.51	
	200	14.82	11.83	1.25		4.84	2.94	1.65	
	250	23.07	16.86	1.37		8.08	4.44	1.82	
AR9	100	5.44	3.78	1.44	1.38	1.84	1.00	1.84	1.85
	150	10.37	7.98	1.30		3.47	2.01	1.73	
	200	16.70	11.83	1.41		5.81	2.94	1.98	
DF1	100	5.23	6.15	0.85	0.87	2.07	2.05	1.01	1.04
	150	9.51	10.85	0.88		3.75	3.52	1.06	
DF4	100	2.92	2.49	1.17	1.22	1.22	0.96	1.27	1.31
	150	6.25	4.94	1.27		2.45	1.82	1.34	
ID1	150	3.09	3.11	0.99	0.94	0.98	1.26	0.78	0.73
	200	5.03	5.46	0.92		1.55	2.18	0.71	
	250	7.25	7.92	0.92		2.25	3.23	0.70	

Table 2: Comparison between Experimental and FEA displacements.

3.3 Load/strains results

This section presents the results obtained for the strains in the top reinforcement bars. In Figures 7 to 9 and Table 3 a comparison between the strains measured in the experimental tests and in the FEA is presented for several load levels (Table 3).

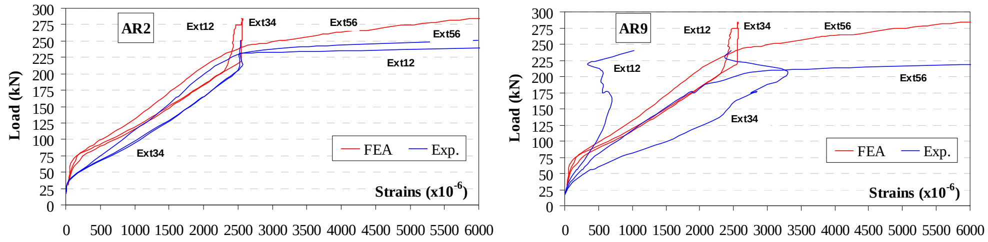

Figure 7: Load/reinforcement strain graphs for models AR2 and AR9.

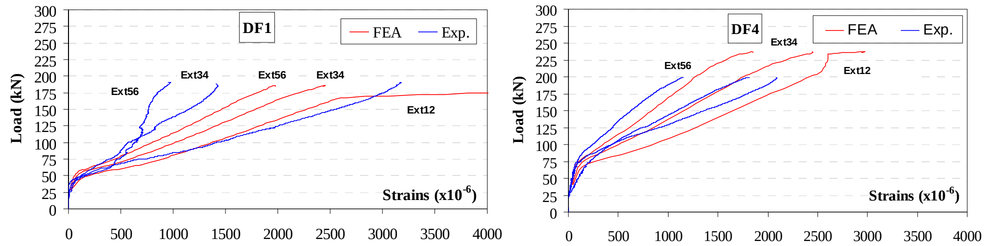

Figure 8: Load/reinforcement strain graphs for models DF1 and DF4.

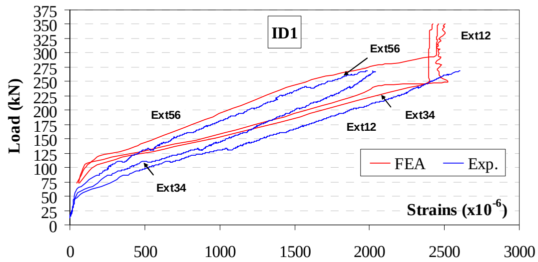

Figure 9: Load/reinforcement strain graphs for model ID1.

From Table 3, it can be stated that the average value of Exp./FEA for all the tested models is 1.07 (it varied between 0.78 for Model DF4 to 1.35 for Model AR2), witch gives a good agreement between the experimental results and the FEA predictions.

Model	Load (kN)	Ext.1+2 ($x10^{-6}$)			Ext.3+4 ($x10^{-6}$)			Ext.5+6 ($x10^{-6}$)		
		Exp.	FEA	Exp/ FEA	Exp.	FEA	Exp/ FEA	Exp.	FEA	Exp/ FEA
AR2	100	1081	573	1.89	1041	643	1.62	822	462	1.78
	150	1799	1454	1.24	1790	1498	1.19	1443	1206	1.20
	200	2447	2231	1.10	2433	2240	1.09	2017	1849	1.09
			Average	1.41		Average	1.30		Average	1.35
AR9	100	462	573	0.81	1523	643	2.37	780	462	1.69
	150	666	1454	0.46	2413	1498	1.61	1469	1206	1.22
	200	551	2231	0.25	3224	2240	1.44	2592	1849	1.40
			Average	0.51		Average	1.81		Average	1.44
DF1	100	1415	1352	1.05	573	1001	0.57	629	781	0.80
	150	2542	2260	1.12	1148	1773	0.65	764	1451	0.52
			Average	1.09		Average	0.61		Average	0.66
DF4	100	434	830	0.52	439	490	0.89	243	338	0.72
	150	1360	1634	0.83	1086	1142	0.95	629	825	0.76
			Average	0.68		Average	0.92		Average	0.74
ID1	150	1276	768	1.66	1046	837	1.25	686	508	1.35
	200	1872	1563	1.20	1500	1667	0.90	1189	1045	1.14
	250	2416	2514	0.96	1943	2395	0.81	1740	1571	1.11
			Average	1.27		Average	0.99		Average	1.20

Table 3: Comparison between Experimental/FEA measured strains.

3.4 Failure loads

All the slabs failed by punching and their ultimate loads (V_{exp}), including self weight, are given in Table 4. Also the experimental and FEA results are compared with the predicted punching resistance quantified using EC2[6] and ACI 318-08[7].

Model	V_{exp} (kN)	V_{FEA} (kN)	V_{exp}/V_{FEA}	Code	V_{Rm} (kN)	V_{exp}/V_{Rm}	V_{FEA}/V_{Rm}
AR2	258	284	0.91	EC2	270	0.96	1.05
				ACI 318-08	187	1.38	1.52
AR9	251	284	0.88	EC2	273	0.92	1.04
				ACI 318-08	187	1.34	1.52
DF1	191	186	1.03	EC2	203	0.94	0.92
				ACI 318-08	138	1.38	1.35
DF4	199	237	0.84	EC2	217	0.92	1.09
				ACI 318-08	167	1.19	1.42
ID1	269	354	0.76	EC2	274	0.98	1.29
				ACI 318-08	196	1.38	1.81

Table 4: Comparison between effective Exp./FEA loads and predicted failure loads.

The limitation of the parameter $\left(1+\sqrt{200/d}\right)$ in EC2[6] to a maximum of 2 was neglected. In the quantification of the punching resistance the mean values of the materials strengths, without partial coefficients, were used. V_{Rm} is the predicted failure load (EC2[6] or ACI 318-08[7]).

The EC2[6] predictions for the failure loads give satisfactory results when compared with the obtained experimental values. The average ratio V_{exp}/V_{Rm} was 0.94. The strength of the slabs predicted using ACI 318-08[7] are highly underestimated, with an average ratio V_{exp}/V_{Rm} of 1.33. The FEA overestimates the failure load, except for model DF1. The average ratio V_{exp}/V_{FEA} obtained was 0.88. The average ratio V_{FEA}/V_{Rm} was 1.08 for EC2[6] and 1.52 for ACI 318-08[7].

5. CONCLUSIONS

The results of the non-linear three-dimensional numerical analysis of this set of flat slab models give a satisfactory agreement with the experimental results. This comparison was obtained in terms of deflections, strains in the top reinforcement bars and of punching resistance. The 3D FEA analysis shows that is it possible to effectively simulate the real behaviour of column/slab connections, with a certain degree of accuracy. One of the most important things in this kind of analysis is the correct choice of the adequate material modelling. It can be assumed that this finite element program can be used for further research of reinforced concrete slab /column connections behaviour.

The EC2[6] preditions for the punching load gives a good agreement with the experimental and FEA values obtained. Otherwise the ACI 318-08[7] is too conservative, it seriously underestimates underestimates the punching resistance.

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REFERENCES

[1] ATENA Program Documentation (2007). Cervenka Consulting, Prague, Czech Republic.

[2] Ramos, A. M. P. and Lúcio, V., *Punching of Prestressed Flat Slabs: Experimental Analysis*, International Workshop on Punching Shear Capacity of RC Flat Slabs, Royal Institute of Technology, Department of Structural Engineering, Estocolmo, June, 2000.

[3] Ramos, A., Duarte, I. and Lúcio, V.; *Strengthening of Flat Slabs with Transverse Reinforcement*, Proceedings of CCC 2008 - Challenges for Civil Construction, FEUP, Porto, April, 2008.

[4] Faria, Duarte, Lúcio, Válter and Ramos, António, *Strengthening of Reinforced Concrete Slabs Using Post-Tensioning with Anchorages by Bonding*, fib Symposium: Concrete: 21st Century SuperHero, London, June, 2009.

[5] Comité Euro-International du Betón (1990). *CEB-FIP Model Code 1990*.

[6] European Committee for Standardization. 2004. *EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings.*

[7] American Concrete Institute: ACI 318-08. 2008. *Building Code Requirements Structural Concrete and Commentary*, ACI Committee 318.